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PROGRESS REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON DESIGN IN LIGHTWEIGHT STRUCTURAL ALLOYS

STRUCTURAL DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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REPORTS

SPECIFICATIONS FOR HEAVY DUTY STRUCTURES OF HIGH-STRENGTH ALUMINUM ALLOY

PROGRESS REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON DESIGN IN LIGHTWEIGHT STRUCTURAL ALLOYS

Synopsis

These specifications cover allowable stresses, design rules, and fabrication procedures for riveted heavy duty structures built of the high-strength aluminum alloy known commercially as 14S–T6. The basic allowable tensile working stress is 22,000 lb per sq in. based on a minimum yield strength of 53,000 lb per sq in. and a minimum tensile strength of 60,000 lb per sq in.

PART I. GENERAL

Introduction

High-strength aluminum alloys having only 36% of the weight of steel per unit of volume can be used advantageously in many structural applications, especially in cases in which the elimination of dead weight is of considerable importance. These specifications cover the allowable stresses, the design rules, and the fabrication procedures for structures built of the high-strength aluminum alloy most commonly used for heavy duty structural purposes. In the preparation of these specifications the Committee has made use of the available theoretical and experimental work relating to this subject and especially to previous specifications by O. H. Ammann, Shortridge Hardesty, and the late Leon S. Moisseiff,[1] Members, ASCE.

These specifications are confined to allowable stresses, design rules, and fabrication. No attempt has been made to cover the loading, erection, inspection, or nontechnical provisions included in many specifications, since such provisions are fairly well established in current good structural practice. Furthermore, no attempt has been made to include design rules which cover every detail of construction but rather those which are different from steel

NOTE.—Please forward all comments on this report directly to Chairman E. C. Hartmann, Box 772, New Kensington, Pa.

[1] "Design Specifications for Bridges and Structures of Aluminum Alloy 27S-T," by Leon S. Moisseiff, Aluminum Co. of America, Pittsburgh, Pa., 1940.

practice or which are needed for the sake of completeness. It is intended, of course, that structures built under these specifications will be designed, constructed, and erected by following the current good practice already well established for steel structures, except as modified herein.

Material

The principal material considered in these specifications is a high-strength aluminum alloy having the following nominal chemical composition:

Composition	Percentage by weight
Copper	4.4
Silicon	0.8
Manganese	0.8
Magnesium	0.4
Aluminum	93.6
Total	100.0

This material is covered by the American Society for Testing Materials (ASTM) Specifications Nos. B221–49T(CS41), B211–49T(CS41), and B209–49T(Clad CS41). It is produced by several manufacturers under the commercial designation 14S–T6 and is available in the form of shapes, tubing, rods, bars, and forgings. It is also produced in the form of sheet and plate covered on both surfaces with an integral coating, or "cladding," of a corrosion-resistant aluminum alloy. In the latter form it is identified commercially by the designations, Alclad 14S–T6 and R–301. All these products are given a solution heat treatment and a precipitation heat treatment before being shipped.

The specified minimum tensile properties of this material are not the same in all the various products (plate, shapes, etc.). The specified minimum tensile strengths vary from 60,000 lb per sq in. to 68,000 lb per sq in., and the specified minimum yield strengths vary from 53,000 lb per sq in. to 58,000 lb per sq in. The following are the lowest of the various specified minimum properties and have been used as a basis for the selection of allowable stresses in these specifications (in pounds per square inch):

Description	Stress
Tensile strength	60,000
Yield strength (offset 0.2%)	53,000

In addition to the specified minimum tensile properties the engineer will be interested in some of the other mechanical properties not covered by specifications. The following are typical mechanical properties of this alloy and may be considered applicable to "nonclad" products, such as shapes, and to "clad" plate:

Shear strength, in pounds per square inch	41,000
Modulus of elasticity in tension and compression, in pounds per square inch	10,600,000
Modulus of elasticity in shear, in pounds per square inch	4,000,000
Poisson's ratio	1/3
Coefficient of expansion per degree Fahrenheit	0.000012
Weight, in pounds per cubic inch	0.101

(The foregoing value of shear strength is typical as determined with steel shearing tools. The value determined by torsion tests is greater.)

Alloy 14S–T6 is the one principally considered in the preparation of these specifications and the one to which the allowable stresses for parts other than rivets and bolts apply. However, these specifications may be applied to structures built of other suitable aluminum alloys, provided such alloys meet the specified strengths and elongations listed in the ASTM specifications mentioned in the first paragraph of this section.

TABLE 1.—Alloys to Be Used for Rivets

Designation before driving	Driving procedure	Designation after driving	Typical shear strength[a]
A17S-T4........	Cold, as received	A17S-T3	33,000
61S-T4........	Hot, 990° F to 1,050° F	61S-T43	24,000

[a] Typical ultimate shear strength of the driven rivet, in pounds per square inch.

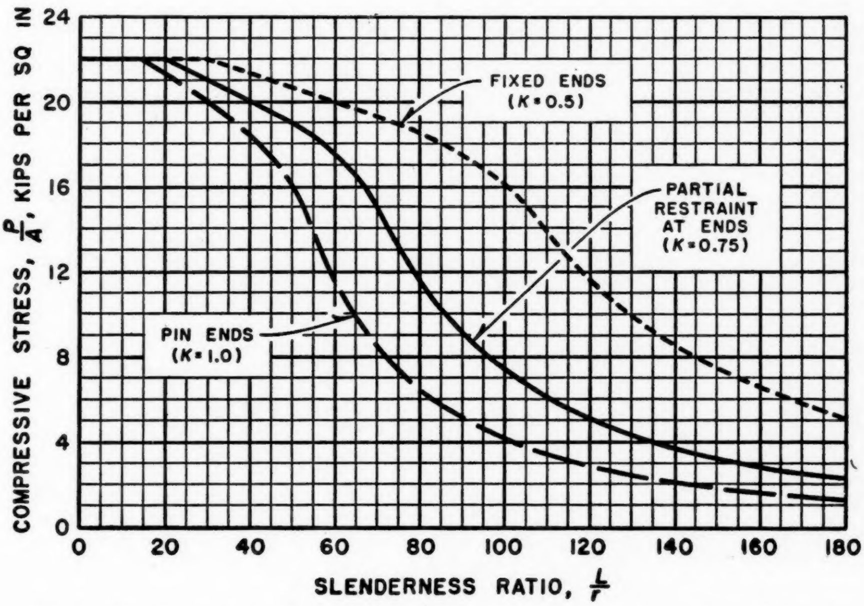

Fig. 1.—Allowable Compressive Stresses for Axially Loaded Columns (Gross Section)

Rivets used in fabricating structures designed in accordance with these specifications shall be of alluminum alloy and may be either cold driven or hot driven. The alloys used are indicated in Table 1. Supplementary information on riveting was published[2] by E. C. Hartmann, M. ASCE, G. O. Hoglund, and M. A. Miller in 1944.

[2] "Joining Aluminum Alloys," by E. C. Hartmann, G. O. Hoglund, and M. A. Miller, *Steel*, August 7, 1944, p. 84.

Permanent bolts used in structures designed in accordance with these specifications shall be of the aluminum alloy known commercially as 24S–T4. Such bolts have a specified minimum ultimate shear strength of 37,000 lb per sq in.

The materials covered by these specifications are heat treated for maximum strength. They cannot be welded without a considerable loss in strength. Structures designed under these specifications shall be assembled by riveting or bolting.

PART II. SPECIFICATIONS

Section A. Summary of Allowable Stresses

The allowable stresses to be used in proportioning the parts of a structure shall be as follows:

Specification	Description	Stress in pounds per square inch
A–1	Axial tension, net section (see Specification H–4)	22,000
A–2	Tension in extreme fibers, of rolled shapes, girders, and built-up members subject to bending, net section (see Specification H–4)	22,000
A–3	Axial compression (see Section B)	....
A–4	Compression in extreme fibers of rolled shapes, girders, and built-up members subject to bending (see Section C)	....
A–5	Compression in plates, legs, and webs (see Section D)	....
A–6	Stress in extreme fibers of pins	34,000
A–7	Shear in plates and webs (see Section E)	....
A–8	Shear in aluminum alloy A17S–T3 rivets, cold driven (see, subsequently, in Tables 5 and 6)	10,000
A–9	Shear in aluminum alloy 61S–T43 rivets, driven at temperatures of from 990° F to 1,050° F (see, subsequently, in Tables 5 and 7)	8,000
A–10	Shear in turned bolts of aluminum alloy 24S–T4 in reamed holes (see, subsequently, in Table 5)	12,000
A–11	Shear in pins	16,000
A–12	Bearing on pins	30,000
A–13	Bearing on hot-driven or cold-driven rivets, milled stiffeners, turned bolts in reamed holes, and other parts in fixed contact (see Section G)	36,000

Section B. Column Design

B–1. Allowable Compressive Stress in Columns.—The allowable compressive stress on the gross section of axially loaded columns shall be determined from the curves in Fig. 1. Let k be a factor describing end restraint. Ordinarily the curve for partial restraint ($k = 0.75$) shall be used. The curves for pin-ended and fixed-ended columns, also shown in Fig. 1, may be used as a guide in the selection of allowable compressive stresses for those cases in which the degree of end restraint is known to be different from that represented by

$k = 0.75$. It is important, however, that no allowable stresses higher than those given for the case of $k = 0.75$ be used in actual design unless a detailed analysis of the structure demonstrates convincingly that a value of k smaller than 0.75 is justified for the member in question.

Columns having cross sections involving webs and outstanding legs of such proportions that local buckling may control the design shall be checked by the method outlined in Section D.

B–2. Maximum Slenderness Ratio.—The ratio of unsupported length to least radius of gyration for compression members shall not exceed 120.

B–3. Connections.—Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

B–4. Compression Splices.—Members designed for compression, if faced for bearing, shall be spliced on four sides sufficiently to hold the abutting parts true to place. The splice shall be as near a panel point as practicable and shall be designed to transmit at least one half of the stress through the splice material. Members not faced for bearing shall be fully spliced for the computed stress. In either case, adequate provision shall be made for transmitting shear.

B–5. Stay Plates.—On the open sides of compression members, the flanges shall be connected by lacing bars, and there shall be stay plates as near each end as practicable. There shall be stay plates at intermediate points where the lacing is interrupted. In main members the length of the end stay plates shall not be less than one and one-fourth times the distance between rivet lines. The thickness of stay plates shall not be less than one fortieth of the distance between rivet lines.

B–6. Diagonal Lacing.—The slenderness ratio of the part of the flange between the lacing bar connections shall be not more than two thirds of the slenderness ratio of the member.

The angle between the center line of the lacing bar and the center line of the column shall be about 60° for single lacing and about 45° for double lacing.

B–7. Combined Compression and Bending.—The allowable bending stress in a member which carries bending moment in addition to uniform compression (as, for example, an eccentrically loaded column) shall be determined from one of the following two formulas—whichever gives the lower value:

a. Failure by Bending in Plane of Bending Forces.—The maximum bending stress (compression) which may be permitted at or near the center of the unsupported length, in addition to uniform compression, P/A, equals

$$f_b = \left(f_B - \frac{P}{A}\right)\left(1 - \frac{P/A}{f_C}\right) \qquad (1)$$

in which (in pounds per square inch):

P/A is the average compressive stress on the gross cross section (A) of a member, produced by a column load, P;

f_B is the allowable compressive working stress for a member considered as a beam (see Section C); and

f_C is the allowable working stress for a member considered as an axially loaded column tending to fail in the plane of the bending forces.

b. Failure by Buckling Normal to Plane of Bending Forces.—The maximum bending stress (compression) which may be permitted at or near the center of the unsupported length, in addition to the uniform compression, P/A, equals

$$f_b = f_B\left(1 - \frac{P/A}{f_{C_n}}\right) \quad \text{(2)}$$

in which f_{C_n} is the allowable working stress for a member considered as an axially loaded column tending to fail in a direction normal to the plane of bending forces, in pounds per square inch.

B–8. Transverse Shear in Columns.—In designing lacing or shear webs for columns, the maximum shear on the column shall be computed from the formula:

$$V = P\,\frac{4.5\,r^2\,(f_B - P/A)}{f_C\,L\,c} + V_t \quad \text{(3a)}$$

but shall not be taken less than

$$V = 0.02\,P + V_t \quad \text{(3b)}$$

in which:

V is the maximum transverse shear on any transverse section of a column in the outer eighth of the length at each end, in the direction of assumed bending, in pounds;

r is the radius of gyration, in inches;

L is the length of the member, in inches;

c is the distance from the centroidal axis to the extreme fiber, in inches; and

V_t is the shear due to any transverse loads on a column, in pounds.

The values of f_B, f_C, L, r, and c must be consistent with the direction of bending assumed.

Section C. Allowable Compressive Stresses in Flanges of Beams and Girders

C–1.—The allowable compressive stress in the extreme fiber (gross section) of single-web rolled shapes, extruded shapes, girders, and built-up sections, subject to bending, shall be determined from the curve in Fig. 2. The terms used in Fig. 2 are defined as follows:

L is the laterally unsupported length of compression flange (clear distance between supports at which the beam is prevented from lateral displacement), in inches;

S_c is the section modulus for the beam about the axis normal to the web (compression side), in inches;

$$B = I_1\,d\sqrt{11.7 + \frac{J}{I_1}\left(\frac{L}{d}\right)^2};$$

I_1 is the moment of inertia for the beam about the axis parallel to the web, in inches^4;

J is the torsion factor, in inches^4; and

d is the depth of beam, in inches.

In the case of beams having top and bottom flanges of different lateral stiffness, I_1 should be calculated as if both flanges were the same as the compression

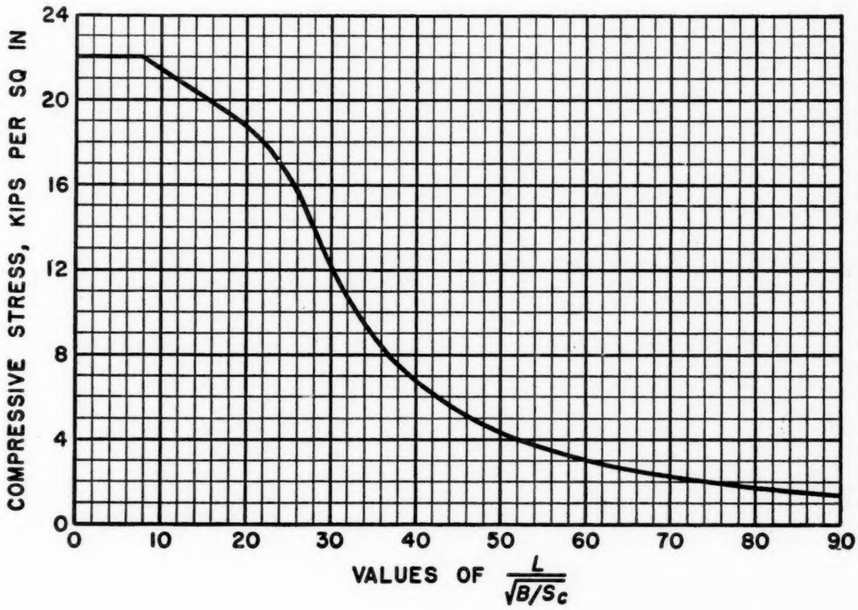

FIG. 2.—ALLOWABLE COMPRESSIVE STRESSES IN BEAM AND GIRDER FLANGES (GROSS SECTION)

flange. Values of the torsion factor J are published for many standard shapes.[3] Values of J for plates and shapes not published may be calculated by assuming the section to be composed of rectangles and taking the sum of the terms $b\ t^3/3$ for each rectangle, in which b equals the length and t, the thickness of the rectangle, both in inches. The value of J for a built-up member is the sum of the individual values of J of the sections of which it is composed.

The allowable stresses from Fig. 2 provide a safe margin against the lateral buckling type of failure. The outstanding compression flanges of the beams and girders should be checked for local buckling by the method outlined in Section D.

Table 2 lists values of allowable stress determined from Fig. 2 and Section D for various laterally unsupported lengths of a number of standard I-beams and H-beams. Table 3 lists similar values for standard channels.

Because of their tube-like cross section, double-web box girders are very stiff in torsion compared with single-web girders of comparable size, and,

[3] "Alcoa Structural Handbook," Aluminum Co. of America, Pittsburgh, Pa., 1950, pp. 87–112.

hence, lateral buckling failures such as are considered in Fig. 2 do not occur in such girders. For double-web box girders it is necessary only to check for local buckling of the flanges by the method outlined in Section D.

Section D. Allowable Compressive Stress for Plates, Legs, and Webs

D-1.—For struts consisting of a single angle or a T-section the compressive stress on the gross area shall not exceed the values given by the curves in Fig. 3 or Fig. 1, whichever is smaller.

TABLE 2.—Allowable Compressive Stress in Beam Flanges for Various Values of Laterally Unsupported Length of Compression Flange, L, in Inches

Procedure.—Maximum allowable bending moments are found by multiplying the allowable compressive stresses (in pounds per square inch) by the gross section modulus of the beam. The stress on the net section of the tension flange must also be kept within allowable limits.

Depth (in.)	Weight (lb per ft)	Section modulus (in.^{3})	Values of L									
			16	32	64	96	128	160	192	256	352	480
(a) I-Beams												
2	0.804	0.481	20,600[a]	18,800	10,200	6,100	4,400	3,500	2,900	2,100	1,600	....
2	1.473	0.782	21,600	19,900	17,000	12,200	9,100	7,200	6,000	4,500	3,300	2,400
2.5	1.850	1.162	21,800	20,300	18,300	15,600	12,000	9,600	8,000	6,000	4,400	3,200
3	2.02	1.68	21,400	19,800	15,800	10,200	7,500	5,900	5,000	3,700	2,700	2,000
3	2.67	1.95	21,700	20,300	17,900	14,100	10,400	8,300	6,900	5,200	3,800	2,800
4	2.72	3.03	21,700	20,000	15,700	9,700	7,100	5,500	4,600	3,400	2,500	1,800
4	3.74	3.59	21,800[a]	20,400	17,800	13,600	10,000	7,900	6,500	4,900	3,500	2,600
5	3.53	4.90	21,800[a]	20,200	16,200	9,900	7,100	5,500	4,500	3,400	2,400	1,800
5	5.25	6.09	21,900[a]	20,600	18,200	14,700	10,800	8,500	7,000	5,200	3,800	2,800
6	4.43	7.36	21,800[a]	20,500	16,900	10,500	7,300	5,600	4,600	3,400	2,400	1,800
6	6.13	8.77	21,800[a]	20,700	18,000	13,600	9,800	7,600	6,300	4,600	3,400	2,500
7	5.42	10.48	21,800[a]	20,800	17,500	11,200	7,700	5,900	4,700	3,400	2,500	1,800
7	7.12	12.12	21,800[a]	20,900	18,200	13,400	9,400	7,300	5,900	4,400	3,100	2,300
8	6.53	14.39	21,800[a]	21,000	18,000	12,200	8,200	6,100	5,000	3,600	2,500	1,800
8	9.07	17.18	21,800[a]	21,100	18,700	15,000	10,500	8,000	6,600	4,800	3,400	2,500
9	7.72	19.09	21,800[a]	21,200	18,500	13,200	8,800	6,500	5,200	3,700	2,600	1,900
9	10.68	22.75	21,800[a]	21,200	18,900	15,600	10,900	8,300	6,700	4,900	3,500	2,500
10	9.01	24.68	21,800[a]	21,300	18,800	14,500	9,400	6,900	5,500	3,900	2,700	2,000
10	12.45	29.41	21,800[a]	21,400	19,200	16,300	11,500	8,700	7,000	5,100	3,600	2,600
12	11.31	36.35	21,800[a]	21,500	19,100	15,200	9,800	7,100	5,600	3,900	2,700	1,900
12	17.78	50.81	22,000	21,700	19,800	17,600	14,000	10,400	8,400	6,000	4,200	3,100
(b) H-Beams												
4	4.85	5.36	21,700[a]	21,400[a]	19,600	17,700	15,100	11,800	9,700	7,200	5,200	3,800
5	6.63	9.53	21,500[a]	21,500[a]	20,100	18,500	16,400	13,100	10,600	7,700	5,500	4,000
6	8.04	14.69	21,300[a]	21,300[a]	20,500	19,000	17,200	14,400	11,400	8,000	5,600	4,000
6	9.40	15.81	21,300[a]	21,300[a]	20,600	19,200	17,700	15,600	12,700	9,000	6,300	4,600
8	11.51	28.23	20,700[a]	20,700[a]	20,400[a]	19,700[a]	18,600	16,700	13,700	9,100	6,000	4,200
8	13.32	30.23	20,700[a]	20,700[a]	20,400[a]	19,800[a]	18,800	17,200	14,800	10,000	6,700	4,700

[a] These values are governed by local buckling (see Section D). All other values are determined from Fig. 2, Section C.

D-2.—For compression members other than those consisting of a single angle or a T-section the following procedure shall be followed to provide a suitable margin of safety against the weakening effects of local buckling of flat plates, legs, and webs:

a.—Compute the compressive stress f_c on the flat plate, leg, or web in question, based on the design loads and the gross area, without regard to local buckling. This stress must be within allowable limits as defined in Sections B and C.

b.—Find the limiting value of b/t corresponding to the stress, f_c, by the use of Fig. 4 or Fig. 5. If the flat plate, leg, or web has a ratio of unsupported width to thickness not exceeding this limiting value, local buckling is not a problem and the full gross area of the plate, leg, or web may be considered effective.

TABLE 3.—ALLOWABLE COMPRESSIVE STRESS IN CHANNEL FLANGES FOR VARIOUS VALUES OF LATERALLY UNSUPPORTED LENGTH OF COMPRESSION FLANGE, L, IN INCHES

Procedure.—Maximum allowable bending moments are found by multiplying the allowable compressive stresses (in pounds per square inch) by the gross section modulus of the beam. The stress on the net section of the tension flange must also be kept within allowable limits.

Depth (in.)	Weight (lb per ft)	Section modulus (in.^{3})	Values of L									
			16	32	64	96	128	160	192	256	352	480
3	1.46	1.10	21,200	19,300	13,200	8,400	6,200	4,900	4,100	3,100	2,200	1,600
3	2.13	1.38	21,600	20,000	17,400	13,200	9,900	7,800	6,500	4,800	3,500	2,600
4	1.90	1.92	21,400	19,200	12,300	7,600	5,500	4,400	3,600	2,700	2,000	1,400
4	2.58	2.29	21,600[a]	19,700	15,700	10,200	7,400	5,900	4,900	3,600	2,600	2,000
5	2.38	3.00	21,600[a]	19,400	12,300	7,400	5,300	4,200	3,400	2,800	1,800	1,300
5	4.09	4.17	21,700[a]	20,200	17,400	12,900	9,500	7,500	6,200	4,600	3,400	2,500
6	2.91	4.37	21,500[a]	19,700	12,900	7,400	5,200	4,100	3,300	2,400	1,800	1,300
6	4.63	5.80	21,700[a]	20,200	16,700	11,100	8,000	6,300	5,200	3,900	2,800	2,000
7	3.47	6.08	21,600[a]	20,000	14,000	7,800	5,400	4,100	3,400	2,500	1,800	1,300
7	6.13	8.64	21,700[a]	20,400	17,500	12,500	9,000	7,000	5,800	4,300	3,100	2,200
8	4.38	8.46	21,700[a]	20,200	15,300	8,400	5,700	4,300	3,500	2,600	1,800	1,300
8	6.99	11.34	21,700[a]	20,500	17,500	12,200	8,700	6,700	5,500	4,100	2,900	2,100
9	4.74	10.60	21,600[a]	20,400	16,000	8,800	5,800	4,400	3,500	2,500	1,800	1,300
9	8.90	15.75	21,700[a]	20,800	18,100	13,700	9,700	7,400	6,200	4,500	3,200	2,400
10	5.43	13.47	21,600[a]	20,600	16,600	9,300	6,100	4,500	3,600	2,500	1,800	1,300
10	10.67	20.69	21,700[a]	21,000	18,500	14,800	10,400	8,000	6,500	4,800	3,400	2,500
12	7.63	21.97	21,600[a]	21,000	17,900	11,300	7,200	5,200	4,100	2,900	2,000	1,400
12	12.45	29.94	21,700[a]	21,100	18,600	14,600	9,900	7,500	6,000	4,400	3,100	2,200

[a] These values are governed by local buckling (see Section D). All other values are determined from Fig. 2, Section C.

c.—If the flat plate, leg, or web has a ratio of unsupported width to thickness greater than the limiting $\frac{b}{t}$-ratio found in step b, only a part of its unsupported width shall be included in computing its effective area. The part of the unsupported width of any individual flat plate, leg, or web which may be considered effective shall be found as follows:

$$b_e = b\frac{f_1}{f_c} \qquad (4)$$

in which:

b_e is that part of the unsupported width considered effective, in inches;
b is the unsupported width, in inches;

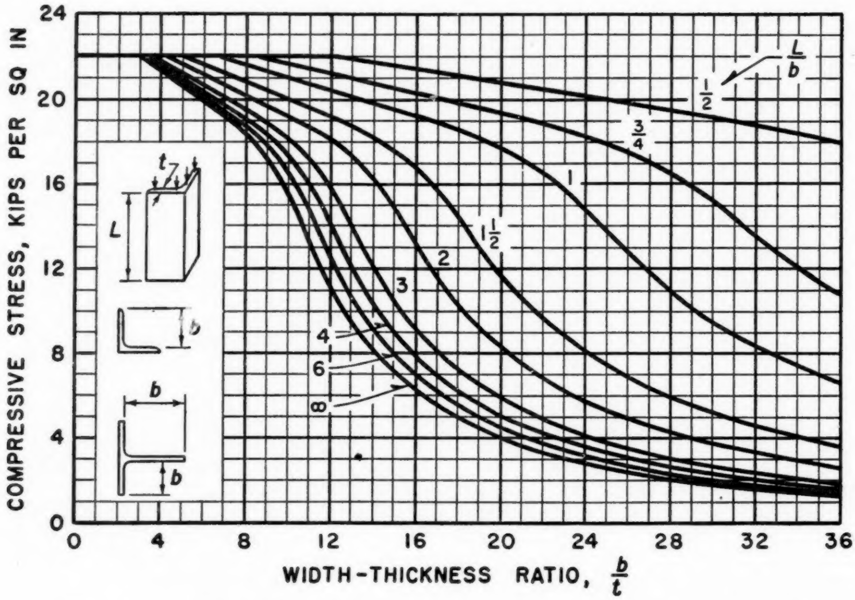

FIG. 3.—ALLOWABLE COMPRESSIVE STRESSES IN OUTSTANDING LEGS OF SINGLE-ANGLE AND T-SECTION STRUTS (GROSS SECTION)

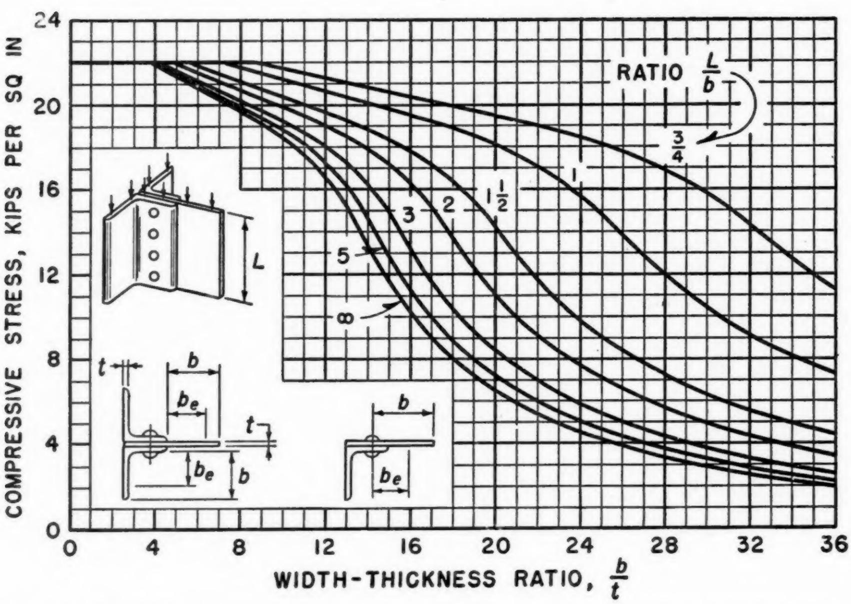

FIG. 4.—CHART FOR DETERMINING EFFECTIVE WIDTH FOR OUTSTANDING LEGS OF ANGLES BUILT INTO OTHER PARTS AND FOR PLATES BUILT IN ALONG ONE EDGE

f_c is the compressive stress based on gross area from step (*a*), in pounds per square inch; and

f_1 is the stress found from Fig. 4 or Fig. 5 corresponding to the $\frac{b}{t}$-value for the plate, leg, or web in question, in pounds per square inch.

d.—Compute the compressive stress on the effective area. In the case of an axially loaded column this is simply the axial load divided by the total effective area, which, in turn, is simply the sum of the effective areas of the component parts. In the case of a beam or girder the compressive stress on the effective area shall be determined as follows: Compute the compressive extreme fiber stress f_c for the gross section of the beam or girder and then multiply

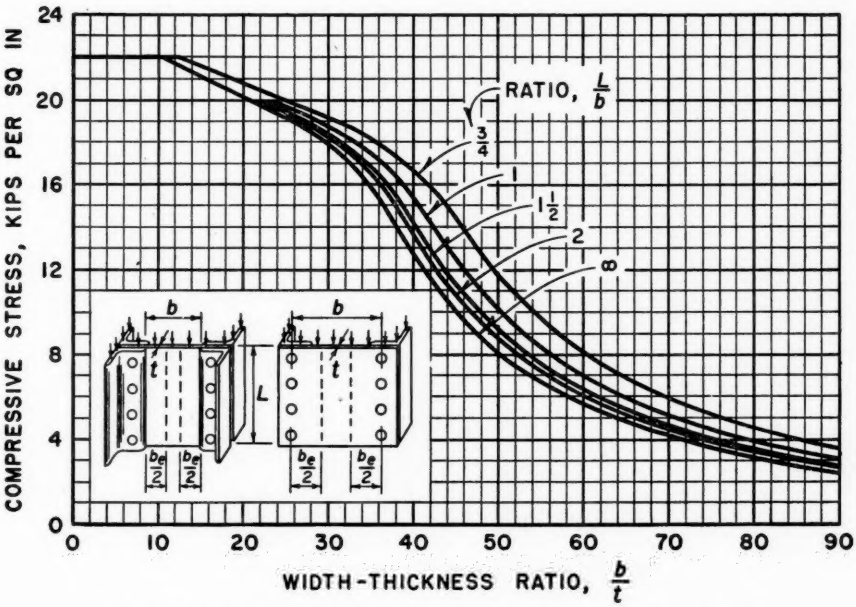

FIG. 5.—CHART FOR DETERMINING EFFECTIVE WIDTH FOR FLAT PLATES BUILT IN ALONG TWO EDGES

this value by the ratio of the gross compression flange area to the effective compression flange area, including in both flange areas not only the flange proper but also that part of the web in the outermost one sixth of the over-all depth of the beam or girder.

e.—The compressive stress on the effective area computed in accordance with step d shall not exceed allowable limits as defined in Section B and C for the gross area.

f.—Step c provides a suitable factor of safety against the collapse of the member as a whole but does not necessarily provide complete protection against the local buckling of individual flat surfaces at the design load. Where local buckling at the design load cannot be tolerated because of appearance, or

for other reasons, the compressive stress on the gross section of the member shall be less than 1.5 times the value given in Fig. 4 or Fig. 5 for the $\frac{b}{t}$-ratio in question.

Section E. Allowable Shear Stresses in Plates and Webs

E-1.—The allowable shear stress on flat webs shall not exceed the values given by the curves in Fig. 6. The values in Fig. 6 apply to the gross area of the web, but the shear on the net area shall not exceed 15,000 lb per sq in.

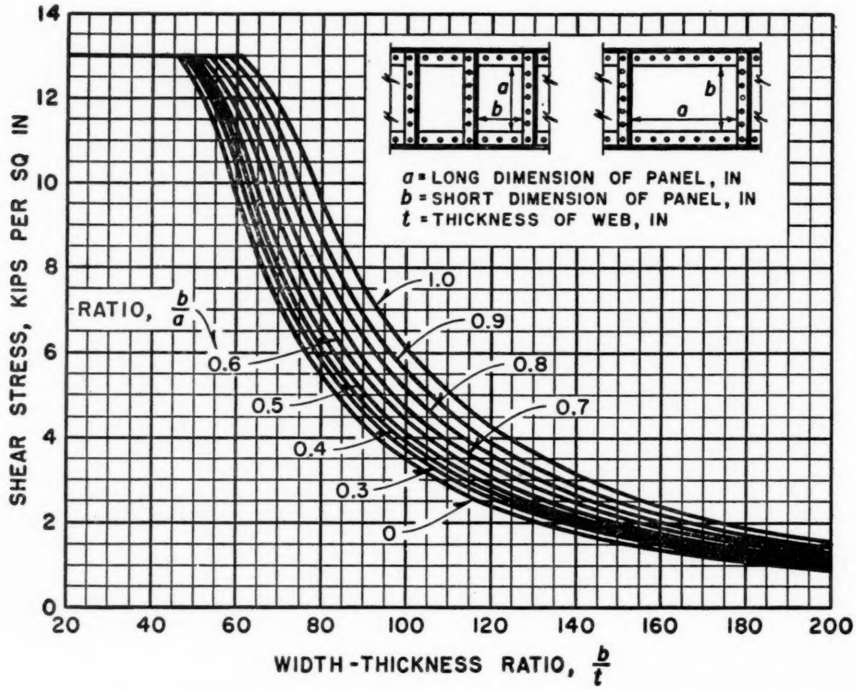

Fig. 6.—Allowable Shear Stresses on Webs; Partial Restraint Assumed at Edges of Rectangular Panels (Gross Section).

Section F. Plate Girder Design

F-1. Proportioning Plate Girders.—Plate girders shall be proportioned by the moment of inertia method, with the gross section used to determine the moment of inertia.

The stress on the net area of the tension flange shall be found by multiplying the stress on the gross section by the ratio of the gross area of the tension flange to the net area. In determining this ratio the tension flange shall be considered to consist of the flange angles and cover plates plus that part of the web included in the outermost one sixth of the over-all height of the girder.

F-2. Allowable Flange Stress.—The allowable compressive stress in the extreme fiber of plate girders shall be determined as outlined in Sections C

and D. The numerical value of the term $\sqrt{B/S_c}$, used in Fig. 2, is rarely less than one half of the width, in inches, of the compression flange for a plate girder. This fact is useful in preliminary design.

F–3. Flange Cover Plates.—Cover plates shall be equal in thickness, or shall diminish in thickness, from the flange angles outward. No plate shall be thicker than the flange angles. Cover plates shall extend far enough to allow at least two extra rivets at each end of the plate beyond the theoretical end, and the spacing of the rivets in the remainder of the plate shall be such as to develop the required strength of the plate at any section.

F–4. Flange Rivets.—The flanges of plate girders shall be connected to the web with enough rivets to transmit the horizontal shear at any point together with any load that is applied directly on the flange.

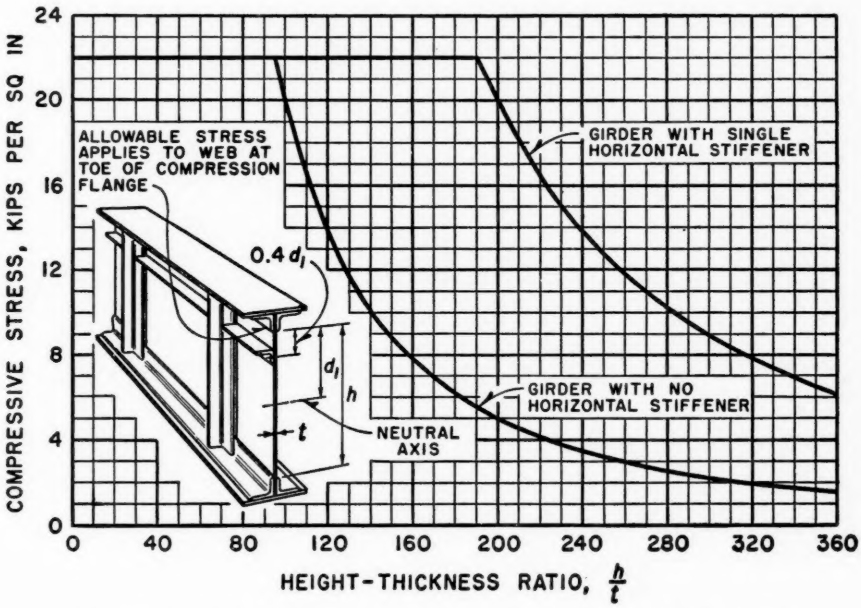

FIG. 7.—ALLOWABLE LONGITUDINAL COMPRESSIVE STRESSES FOR WEBS OF GIRDERS

F–5. Flange Splices.—It is preferable that flange angles be spliced with angles and that no two members be spliced at the same cross section.

F–6. Allowable Web Stresses.—The allowable shear stress in the webs of plate girders shall not exceed the values given by the curves in Fig. 6. The longitudinal compressive stress in webs of plate girders at the toe of the compression flange shall not exceed the values given by the curves in Fig. 7.

F–7. Web Splices.—It is preferable that splices in the webs of plate girders be made with splice plates on both sides of the web.

F–8. Spacing of Vertical Stiffeners to Resist Shear Buckling.—The distance between vertical stiffeners shall not exceed the values given by the solid curves in Fig. 8, which are replots of the curves in Fig. 6. The maximum value of

the ratio of stiffener spacing to height of web, s/h, in Fig. 8 shall be determined from the ratio of clear height to thickness, h/t, and the computed shear stress on the girder web. Where a stiffener is composed of a pair of members, one on each side of the web, the distance s shall be the clear distance between the stiffeners. Where a stiffener is composed of a member on one side of the web only, the distance s shall be the distance between rivet lines. In determining the spacing of vertical stiffeners to resist shear buckling in panels containing a horizontal stiffener located as shown in Fig. 7, the distance h in Fig. 8 may be taken as 90% of the clear height between flanges.

F–9. Size of Vertical Stiffeners to Resist Shear Buckling.—Stiffeners applied to plate girder webs to resist shear buckling shall have a moment of inertia not less than the values given by the dotted curves in Fig. 8. The minimum

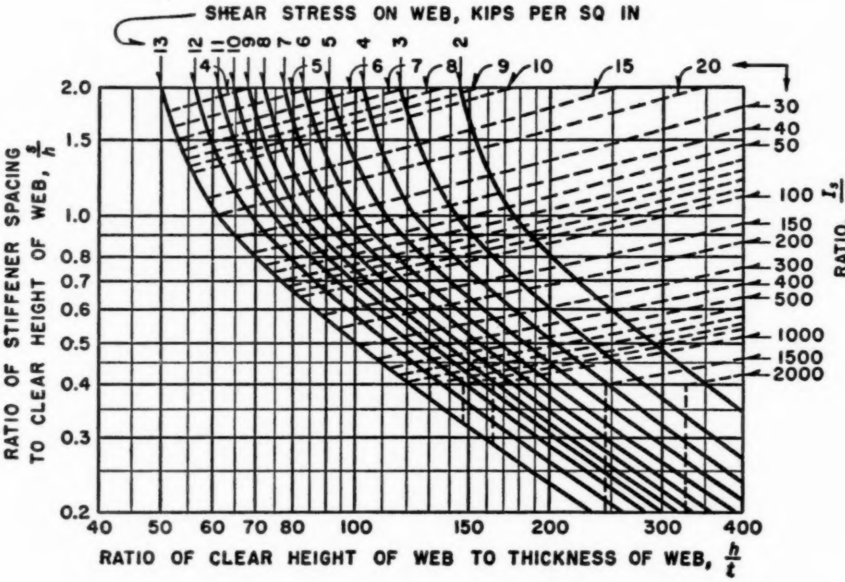

FIG. 8.—SPACING AND MOMENT OF INERTIA OF VERTICAL STIFFENERS TO RESIST SHEAR BUCKLING ON WEBS OF PLATE GIRDERS

value of the ratio of the stiffener moment of inertia to the fourth power of the web thickness, I_s/t^4, in Fig. 8, shall be determined from the ratio of height of web to thickness of web, h/t, and the computed shear stress on the girder web.

For a stiffener composed of members of equal size on both sides of the web, the moment of inertia shall be taken about the center line of the web. For a stiffener composed of a member on one side only, the moment of inertia shall be taken about the face of the web in contact with the stiffener. In determining moment of inertia of stiffeners, the term h shall always be taken as the full clear height between flanges, regardless of whether or not a horizontal stiffener is present.

F–10. Vertical Stiffeners at Points of Bearing.—Stiffeners shall be placed in pairs at end bearings of plate girders and at points of bearing of concentrated

loads. They shall be connected to the web by enough rivets to transmit the load. Such stiffeners shall have a close bearing against the loaded flanges. Only that part of the stiffener cross section which lies outside the fillet of the flange angle shall be considered effective in bearing.

TABLE 4.—VALUES OF STIFFENER COEFFICIENT C_r OF HORIZONTAL STIFFENERS FOR WEBS OF PLATE GIRDERS REINFORCED BY ONE HORIZONTAL STIFFENER

$\frac{s}{h}$	Web Thicknesses, in Inches								
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
0.20	0.18	0.26	0.34	0.43	0.52	0.61	0.71	0.81	0.92
0.25	0.37	0.51	0.67	0.84	1.02	1.21	1.41	1.61	1.86
0.30	0.52	0.73	0.95	1.19	1.45	1.72	2.00	2.29	2.61
0.35	0.65	0.92	1.20	1.50	1.82	2.16	2.51	2.87	3.26
0.40	0.76	1.09	1.43	1.78	2.15	2.54	2.94	3.35	3.78
0.45	0.87	1.25	1.64	2.03	2.45	2.87	3.31	3.76	4.23
0.50	0.97	1.40	1.83	2.26	2.71	3.16	3.63	4.11	4.61
0.55	1.07	1.54	2.00	2.46	2.94	3.42	3.92	4.43	4.96
0.60	1.16	1.67	2.16	2.65	3.16	3.66	4.19	4.73	5.28
0.65	1.25	1.79	2.29	2.83	3.35	3.88	4.43	4.99	5.57
0.70	1.34	1.90	2.42	2.99	3.53	4.08	4.66	5.24	5.83
0.75	1.43	2.00	2.55	3.14	3.71	4.28	4.89	5.48	6.08
0.80	1.51	2.09	2.68	3.29	3.89	4.48	5.11	5.70	6.32
0.85	1.58	2.18	2.80	3.44	4.07	4.67	5.32	5.92	6.55
0.90	1.65	2.27	2.92	3.58	4.24	4.86	5.55	6.13	6.76
0.95	1.72	2.36	3.04	3.72	4.40	5.04	5.71	6.33	6.96
1.00	1.79	2.45	3.15	3.85	4.55	5.22	5.90	6.52	7.15
1.05	1.86	2.54	3.26	3.97	4.69	5.39	6.08	6.70	7.33
1.10	1.93	2.63	3.36	4.09	4.83	5.55	6.25	6.87	7.51
1.15	1.99	2.72	3.46	4.21	4.96	5.70	6.41	7.03	7.68
1.20	2.05	2.80	3.56	4.33	5.09	5.84	6.56	7.19	7.85

The moment of inertia of the stiffener shall not be less than that given by the formula:

$$I = I_s + \frac{P\,h^2}{74{,}000{,}000} \qquad \text{(5)}$$

in which:

I_s is the moment of inertia, in inches^4, required to resist shear buckling (Fig. 8);

P is a local load concentration on the stiffener, in pounds; and

h is the clear height of the web between flanges, in inches.

F-11. Horizontal Stiffeners.—A horizontal stiffener of the type shown in Fig. 7 shall have a radius of gyration r not less than that given by the formula:

$$r = C_r \left(\frac{h}{t}\right)^2 f \times 10^{-9} \qquad \text{(6)}$$

in which:

r is the required radius of gyration of one stiffener about the appropriate axis, in inches;

h is the clear height of web between flanges, in inches;

t is the thickness of web, in inches;

f is the compressive stress at the toe of the flange angles, in pounds per square inch; and

C_r is a coefficient which depends on the ratio of the spacing of the vertical stiffeners s to the clear height of the web, h. Values of C_r are given in Table 4.

For a stiffener composed of members of equal size on both sides of the web, the radius of gyration shall be taken about the center line of the web. In the case of a stiffener consisting of a member on one side only, the radius of gyration shall be taken about the face of the web in contact with the stiffener.

Section G. Riveted and Bolted Connections

G–1. Allowable Loads.—The allowable loads on rivets and bolts shall be calculated using the allowable shear and bearing stresses listed in Section A with the following exceptions:

a.—If a rivet or a bolt is used in relatively thin plates or shapes the allowable shear stress shall be reduced in accordance with the information given[2] in Table 5.

TABLE 5.—Percentage Reduction in Shear Strength of Aluminum Alloy Rivets Resulting from Their Use in Thin Plates and Shapes

Ratio,[a] $\frac{D}{t}$	Loss in double shear[b]	Ratio,[a] $\frac{D}{t}$	Loss in double shear[b]	Ratio,[a] $\frac{D}{t}$	Loss in:		Ratio,[a] $\frac{D}{t}$	Loss in:	
					Single shear	Double shear		Single shear	Double shear
(1)	(3)	(1)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
1.5	0	2.2	9.1	2.9	0	18.2	3.5	2.0	26.0
1.6	1.3	2.3	10.4	3.0	0	19.5	3.6	2.4	27.3
1.7	2.6	2.4	11.7	3.1	0.4	20.8	3.7	2.8	28.6
1.8	3.9	2.5	13.0	3.2	0.8	22.1	3.8	3.2	29.9
1.9	5.2	2.6	14.3	3.3	1.2	23.4	3.9	3.6	31.2
2.0	6.5	2.7	15.6	3.4	1.6	24.7	4.0	4.0	32.5
2.1	7.8	2.8	16.9	....	....	....	....	....	....

[a] Ratio of the rivet diameter, D, to the plate thickness, t. The thickness used is that of the thinnest plate in a single shear joint or of the middle plate in a double shear joint. [b] The percentage loss of strength in single shear is zero for D/t less than 3.0.

b.—If the distance from the center of a rivet or bolt to the edge of a plate or shape toward which the pressure of the rivet or bolt is directed is less than twice the diameter of the rivet or bolt, the allowable bearing stress shall be reduced in accordance with the following:

Ratio of edge distance to rivet or bolt diameter	Allowable bearing stress, in pounds per square inch
2 or more	36,000
$1\frac{3}{4}$	33,000
$1\frac{1}{2}$	30,000

The allowable loads calculated for cold-driven A17S–T3 rivets are given in Table 6 and those for 61S–T43, hot-driven rivets, are given in Table 7.

G–2. Effective Diameter.—The effective diameter of rivets shall be taken as the hole diameter but shall not exceed the values of hole diameter given in Table 6 for cold-driven rivets and in Table 7 for hot-driven rivets. The effective diameter of pins and bolts shall be the nominal diameter of the pin or bolt.

G–3. Bearing Area.—The effective bearing area of pins, bolts, and rivets shall be the effective diameter multiplied by the length in bearing; except that for countersunk rivets, half of the depth of the countersink shall be deducted from the length.

G–4. Arrangement and Strength of Connections.—Connections shall be arranged to minimize the eccentricity of loading on the member. Members and connections shall be proportioned to take into account any eccentricity of loading introduced by the connections.

G–5. Net Section.—The net section of a riveted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width. The net width for a chain of holes extending across the part in any straight or broken line shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding $\frac{s^2}{4\,g}$ for each gage space in the chain. In the correction quantity $\frac{s^2}{4\,g}$:

s is the spacing parallel to direction of load (pitch) of any two successive holes in the chain, in inches; and

g is the spacing perpendicular to direction of load (gage) of the same holes, in inches.

The net section of the part is obtained from that chain which gives the least net width. The hole diameter to be deducted shall be the actual hole diameter for drilled or reamed holes.

For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of the gages from the back of the angle, less the thickness.

For splice members, the thickness shall be only that part of the thickness of the member that has been developed by rivets beyond the section considered.

G–6. Effective Sections of Angles.—If an angle in tension is connected on one side of a gusset plate, the effective section shall be the net section of the connected leg plus one half of the section of the outstanding leg unless the outstanding leg is connected by a lug angle. In the latter case the effective section shall be the entire net section of the angle, and there shall be at least two extra rivets in the lug angle beyond the gusset plate.

G–7. Grip of Rivets.—If the grip of rivets carrying calculated stress exceeds four and one-half times the diameter, the number of rivets shall be increased at least 1% for each additional 1/16 in. of grip. If the grip exceeds six times the diameter, special care shall be taken in driving the rivets to insure that the holes will be filled completely.

G–8. Pitch of Rivets in Built-Up Compression Members.—The pitch in the direction of stress shall be such that the allowable stress on the individual outside plates and shapes, treated as columns having a length equal to the rivet pitch in accordance with Fig. 1, exceeds the calculated stress. In no

TABLE 6.—ALLOWABLE DESIGN LOAD, IN POUNDS PER RIVET, (SHEAR, 10,000 LB PER SQ IN. AND

Dimensions, in Inches Rivet diameter Hole diameter Drill size	3/8 0.386 W		7/16 0.453 29/64		1/2 0.516 33/64		9/16 0.578 37/64	
Thickness of plate, or shape, in inches:	RIVET IN SINGLE SHEAR (ss)							
	ss	ds	ss	ds	ss	ds	ss	ds
1/8	1,170	1,740[b]	1,580[c]	2,040[b]	2,010[c]	2,320[b]	2,520[c]	2,600[b]
3/16	1,170	2,190[c]	1,610	2,880[c]	2,090	3,480[b]	2,620	3,900[b]
1/4	1,170	2,340	1,610	3,120[c]	2,090	3,910[c]	2,620	4,740[c]
5/16	1,170	2,340	1,610	3,220	2,090	4,130[c]	2,620	5,040[c]
3/8	1,170	2,340	1,610	3,220	2,090	4,180	2,620	5,250
7/16	....	....	1,610	3,220	2,090	4,180	2,620	5,250
1/2	....	....	....	....	2,090	4,180	2,620	5,250
9/16	....	....	....	....	....	....	2,620	5,250
5/8	....	....	....	....	....	....	....	....
3/4	....	....	....	....	....	....	....	....
7/8	....	....	....	....	....	....	....	....
1	....	....	....	....	....	....	....	....

[a] Assuming distance from center of rivet to edge of member toward which the pressure of the rivet is directed is by bearing. [c] These values are governed by reduced shear strengths as indicated in Table 5. All other values

TABLE 7.—ALLOWABLE DESIGN LOADS, IN POUNDS PER RIVET, (RIVETS DRIVEN AT 990° F TO 1,050° F; SHEAR, 8,000 LB

Dimensions, in Inches: Rivet diameter Hole diameter Drill size	3/8 0.397 X		7/16 0.469 15/32		1/2 0.531 17/32		9/16 0.594 19/32	
Thickness of plate, or shape, in inches:	RIVET IN SINGLE SHEAR (ss)							
	ss	ds	ss	ds	ss	ds	ss	ds
1/8	990	1,600[b]	1,350[b]	2,050[b]	1,700[b]	2,390[c]	2,080[b]	2,670[c]
3/16	990	1,850[b]	1,380	2,470[b]	1,770	3,010[b]	2,220	3,570[b]
1/4	990	1,980	1,380	2,680[b]	1,770	3,310[b]	2,220	4,000[b]
5/16	990	1,980	1,380	2,760	1,770	3,500[b]	2,220	4,260[b]
3/8	990	1,980	1,380	2,760	1,770	3,540	2,220	4,430
7/16	....	....	1,380	2,760	1,770	3,540	2,220	4,430
1/2	....	....	....	....	1,770	3,540	2,220	4,430
9/16	....	....	....	....	....	....	2,220	4,430
5/8	....	....	....	....	....	....	....	....
3/4	....	....	....	....	....	....	....	....
7/8	....	....	....	....	....	....	....	....
1	....	....	....	....	....	....	....	....

[a] Assuming distance from center of rivet to edge of member toward which the pressure of the rivet is directed is by reduced shear strengths as indicated in Table 5. [c] These values are governed by bearing. All other values

case, however, shall the pitch in the direction of stress exceed six times the diameter of the rivets; and for a distance of one and one-half times the width of the member at each end, the pitch in the direction of stress shall not exceed three and one-half times the diameter of the rivets.

G–9. Stitch Rivets.—Where two or more web plates are in contact, there shall be stitch rivets to make them act in unison. In compression members, the pitch of such rivets in the direction of stress shall be determined as outlined in Specification G–8. The gage at right angles to the direction of stress

FOR COLD-DRIVEN A17S–T3 RIVETS IN 14S–T6 STRUCTURES
BEARING, 36,000 LB PER SQ IN.^{a})

5/8 0.641 41/64		3/4 0.766 49/64		7/8 0.891 57/64		1 1.016 1 1/64		Dimensions, in Inches: Rivet diameter Hole diameter Drill size
OR IN DOUBLE SHEAR (ds)								Thickness of plate, or shape, in inches:
ss	ds	ss	ds	ss	ds	ss	ds	
2,880^b	2,880^b	....	....	....	....	....	....	1/8
3,180^c	4,330^b	4,420^c	5,170^b	5,820^c	6,020^b	....	....	3/16
3,230	5,620^c	4,610	6,890^b	6,110^c	8,020^b	7,780^c	9,140^b	1/4
3,230	6,030^c	4,610	8,140^c	6,240	10,020^b	8,040^c	11,430^b	5/16
3,230	6,310^c	4,610	8,620^c	6,240	11,120^c	8,110	13,720^b	3/8
3,230	6,450	4,610	8,970^c	6,240	11,660^c	8,110	14,580^c	7/16
3,230	6,450	4,610	9,220	6,240	12,070^c	8,110	15,160^c	1/2
3,230	6,450	4,610	9,220	6,240	12,390^c	8,110	15,640^c	9/16
3,230	6,450	4,610	9,220	6,240	12,470	8,110	16,000^c	5/8
....	....	4,610	9,220	6,240	12,470	8,110	16,220	3/4
....	....	....	....	6,240	12,470	8,110	16,220	7/8
....	....	....	....	....	....	8,110	16,220	1

not less than twice the nominal rivet diameter (see Specification G-1, exception *b*). ^{b} These values are governed are governed by basic allowable shear stress.

FOR HOT-DRIVEN 61S–T43 RIVETS IN 14S–T6 STRUCTURES
PER SQ IN.; AND BEARING, 36,000 LB PER SQ IN.^{a})

5/8 0.656 21/32		3/4 0.781 25/32		7/8 0.922 59/64		1 1.063 1 1/16		Dimensions, in Inches: Rivet diameter Hole diameter Drill size
OR IN DOUBLE SHEAR (ds)								Thickness of plate, or shape, in inches:
ss	ds	ss	ds	ss	ds	ss	ds	
2,490^b	2,950^c	....	....	....	....	....	....	1/8
2,660^b	4,120^b	3,680^b	5,170^b	4,980^b	6,220^c	....	....	3/16
2,700	4,710^b	3,830	6,170^b	5,230^b	7,910^b	6,820^b	9,590^b	1/4
2,700	5,060^b	3,830	6,770^b	5,340	8,880^b	7,040^b	10,880^b	5/16
2,700	5,290^b	3,830	7,170^b	5,340	9,530^b	7,100	12,040^b	3/8
2,700	5,410	3,830	7,460^b	5,340	9,990^b	7,100	12,770^b	7/16
2,700	5,410	3,830	7,670	5,340	10,340^b	7,100	13,280^b	1/2
2,700	5,410	3,830	7,670	5,340	10,610^b	7,100	13,690^b	9/16
2,700	5,410	3,830	7,670	5,340	10,680	7,100	14,020^b	5/8
....	....	3,830	7,670	5,340	10,680	7,100	14,200	3/4
....	....	....	....	5,340	10,680	7,100	14,200	7/8
....	....	....	....	....	....	7,100	14,200	1

not less than twice the nominal rivet diameter (see Specification G-1, exception *b*). ^{b} These values are governed are governed by basic allowable shear stress.

shall not exceed twenty times the thickness of the outside plates. In tension members the maximum pitch or gage of such rivets shall be twenty times the thickness of the outside plates; and in tension members composed of two angles in contact the pitch of the stitch rivets shall not exceed 10 in.

G–10. Minimum Spacing of Rivets.—The distance between centers of rivets shall not be less than three times the diameter of the rivets.

G–11. Edge Distance of Rivets.—The distance from the center of a rivet to a sheared, sawed, rolled, or planed edge shall be not less than one and one-half times the diameter, except in flanges of beams and channels, where the minimum distance may be one and one-fourth times the diameter. For rivets under computed stress, the distance from the center of the rivet to the edge of the plate or shape toward which the pressure of the rivet is directed should normally be at least twice the nominal diameter of the rivet. In cases where a shorter edge distance must be used, the allowable bearing stress shall be reduced in accordance with Specification G–1, exception b.

The distance from the edge of a plate to the nearest rivet line shall not exceed six times the thickness of the plate.

G–12. Sizes of Rivets in Angles.—The diameter of the rivets in angles whose size is determined by calculated stress shall not exceed one fourth of the width of the leg in which they are driven. In angles whose size is not so determined, 1-in. rivets may be used in 3 1/2-in. legs; 7/8-in. rivets, in 3-in. legs; and 3/4-in. rivets, in 2 1/2-in. legs.

G–13. Extra Rivets.—If splice plates are not in direct contact with the parts which they connect, there shall be rivets on each side of the joint in excess of the number required in the case of direct contact, to the extent of two extra lines for each intervening plate.

If rivets carrying calculated stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by enough additional rivets to distribute the total stress in the member uniformly over the combined section of the member and filler.

Section H. Miscellaneous Design Rules

H–1. Reversal of Load.—Members subject to reversal of load under the passage of live load shall be proportioned as follows: Determine the tensile load and the compressive load and increase each by 50% of the smaller; then proportion the member and its connections so that the allowable stresses given in Sections A to G, inclusive, will not be exceeded by either increased load.

H–2. Slenderness Ratio of Tension Members.—The ratio of unsupported length to least radius of gyration for tension members shall not exceed the value given by the following formula:

$$\frac{L}{r} = 150 + \frac{f_t}{100} \qquad (7)$$

in which f_t is the lowest net section tensile stress to which the member will be subjected in actual service, in pounds per square inch.

H–3. Stay Plates for Tension Members.—Segments of tension members not directly connected to each other shall be stayed together. The length of the stay plate shall be not less than three fourths of the distance between rivet lines of the segments. Stay plates shall be connected to each segment of the tension member by at least three rivets. The distance between stay plates

shall be such that the slenderness ratio of the individual segments does not exceed that given by Eq. 7, Specification H–2.

H–4. Fatigue.—Tests indicate that riveted members designed in accordance with the requirements of these specifications and constructed so as to be free from severe reentrant corners and other unusual stress raisers will safely withstand at least 100,000 repetitions of maximum live load without fatigue failure regardless of the ratio of minimum to maximum load. Where a greater number of repetitions of some particular loading cycle is expected during the life of the structure, the calculated net section tensile stresses for the loading in question shall not exceed the values given by the curves in Fig. 9. When using the curves in Fig. 9 the reversal-of-load rule in Specification H–1 should be ignored. The final member and connections selected, however, shall be strong enough to satisfy the requirements of Specification H–1.

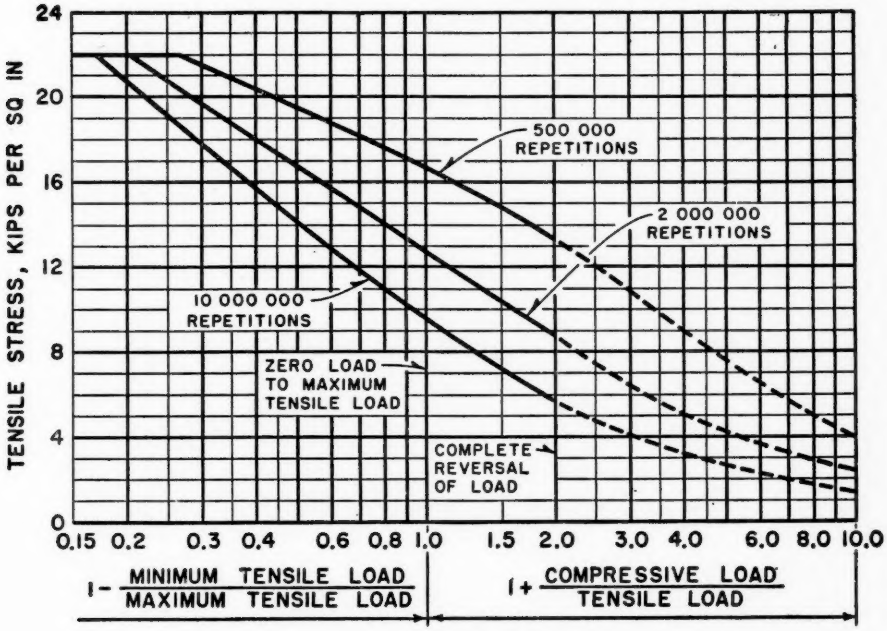

FIG. 9.—ALLOWABLE TENSILE STRESSES ON NET SECTION FOR VARIOUS NUMBERS OF REPETITIONS OF LOAD APPLICATION

In considering fatigue action on structures it is well to bear in mind the following points:

a.—The most severe combination of loadings for which a structure is designed (dead load, maximum live load, maximum impact, maximum wind, etc.) rarely occurs in actual service and is of little or no interest from the standpoint of fatigue.

b.—The loading of most interest from the fatigue standpoint is the steady dead load with a superimposed and repeatedly applied live load having an intensity consistent with day to day normal operating conditions.

c.—The number of cycles of load encountered in structures is usually small compared with those encountered in fatigue problems involving machine parts. It takes many years of service to accumulate even 100,000 cycles of any significant stress application in most structures as is indicated by the following examples: 100,000 cycles represent 10 cycles every day for 27 years; 10,000,000 cycles represent 20 cycles every hour for 57 years. Care must be taken not to overestimate grossly the number of cycles for any given load condition.

d.—Careful attention to details in design and fabrication pays big dividends in fatigue life. When a fatigue failure occurs in a structure it is usually at a point of stress concentration that could have been eliminated with little or no added expense.

Section I. Fabrication

I-1. Laying Out.—

a.—Hole centers may be center punched and cutoff lines may be punched or scribed. Center punching and scribing shall not be used where such marks would remain on fabricated material.

b.—A temperature correction shall be applied where necessary in the layout of critical dimensions. The coefficient of expansion shall be taken as 0.000012 per degree Fahrenheit.

I-2. Cutting.—

a.—Material 1/2 in. thick or less may be sheared, sawed, or cut with a router. Material more than 1/2 in. thick shall be sawed or routed.

b.—Cut edges shall be true and smooth, and free from excessive burrs or ragged breaks.

c.—Edges of plates carrying calculated stresses shall be planed to a depth of 1/4 in. except in the case of sawed or routed edges of a quality equivalent to a planed edge.

d.—Reentrant cuts shall be avoided wherever possible. If used they shall be filleted by drilling prior to cutting.

e.—Flame cutting of aluminum alloys is not permitted.

I-3. Heating.—Structural material shall not be heated, with the following exceptions:

a.—Material may be heated to a temperature not exceeding 400° F for a period not exceeding 15 min to facilitate bending. Such heating shall be done only when proper temperature controls and supervision are provided to insure that the limitations on temperature and time are carefully observed.

b.—Hot-driven rivets shall be heated as specified in Section I-5.

I-4. Punching, Drilling, and Reaming.—Rules for punching, drilling, and reaming are as follows:

a.—Rivet or bolt holes in main members shall be subpunched or subdrilled 3/16 in. smaller than the nominal diameter of the fastener and reamed to

finished size after the parts are firmly bolted together, except that if the metal thickness is greater than the diameter of the hole punching shall not be used.

b.—Rivet or bolt holes in secondary material not carrying calculated stress may be punched or drilled to finished size before assembly.

c.—The finished diameter of holes for cold-driven rivets shall be not more than 4% greater than the nominal diameter of the rivet.

d.—The finished diameter of holes for hot-driven rivets shall be not more than 7% greater than the nominal diameter of the rivet.

e.—The finished diameter of holes for unfinished bolts shall be not more than 1/16 in. larger than the nominal bolt diameter.

f.—Holes for turned bolts shall be drilled or reamed to give a driving fit.

g.—All holes shall be cylindrical and perpendicular to the principal surface. Holes shall not be drifted in such a manner as to distort the metal. All chips lodged between contacting surfaces shall be removed before assembly.

I–5. Riveting.—

a.—The driven head of aluminum alloy rivets preferably shall be of the flat or the low cone type, with dimensions as follows:

(1) Flat heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height not less than 0.4 times the nominal rivet diameter.

(2) Low cone heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height, to the apex of the cone, not less than 0.65 times the nominal rivet diameter. The included angle at the apex of the cone shall be approximately 127°.

b.—Rivets shall be driven hot or cold as called for on the plans, provision for heating being as follows:

(1) Hot-driven rivets shall be heated in a hot air type furnace providing uniform temperatures throughout the rivet chamber and equipped with automatic temperature controls.

(2) Hot-driven rivets shall be held at from 990° F to 1,050° F for not less than 15 min and for not more than 1 hour before driving.

(3) Hot rivets shall be transferred from the furnace to the work and driven with a minimum loss of time.

c.—Rivets shall be driven with direct-acting riveters where practicable.

d.—Rivets shall fill the holes completely. Rivet heads shall be concentric with the rivet holes and shall be in proper contact with the surface of the metal.

e.—Defective rivets shall be removed by drilling.

I–6. Welding.—Welding is not permitted.

I–7. Cleaning and Treatment of Metal Surfaces.—

a.—Surfaces of metal shall be cleaned immediately before painting by a method which will remove all dirt, oil, grease, chips, and other foreign substances.

b.—Either of the two following methods of cleaning may be used on exposed metal surfaces:

(1) Chemical Cleaning.—Parts may be immersed in, or swabbed with, a solution of phosphoric acid and organic solvents diluted with water in the ratio of 1:3. The solution temperature shall be between 50° F and 90° F. The solution shall remain in contact with the metal not less than 5 min. Residual solution shall be removed with clear water.

(2) Sandblasting.—Standard mild sandblasting methods may be used on sections more than 1/8 in. thick.

c.—For contacting surfaces only, the metal may be cleaned in accordance with Specification I–7b, or with a solvent such as mineral spirits or benzine.

d.—Flame cleaning is not permitted.

I–8. Painting.—Specifications to control painting operations are as follows:

a.—Metal parts shall be painted as described in Specifications I–8b to I–8h except where the plans specifically permit a deviation.

b.—Contacting metal surfaces shall be painted before assembly with one coat of zinc chromate primer in accordance with United States Navy Department Specification 52P18 or the equivalent, or with one coat of suitable aluminum pigmented calking compound (brushing consistency with chromate pigment added). Zinc chromate paint shall be allowed to dry before assembly of the parts.

c.—In all cases where aluminum work is to be fastened to steel members or other dissimilar metal parts, the aluminum shall be kept from direct contact with such parts by painting the aluminum surface as described in Specification I–8b and by painting the dissimilar metal with a suitable metal priming paint.

d.—Aluminum surfaces to be placed in contact with concrete or masonry construction shall, before installation, be given a heavy coat of an alkali-resistant bituminous paint. The quality of the bituminous paint used shall be equal to that called for in the Army-Navy Aeronautical Specification AN–P–31. The paint shall be applied as it is received from the manufacturer without the addition of any thinner.

e.—All other surfaces shall be given one shop coat of zinc chromate primer made in accordance with Navy Department Specification 52P18, or one giving equivalent performance.

f.—All surfaces, except those covered by Specifications I–8b, I–8c, and I–8d, shall be given a second shop coat of paint consisting of 2 lb of aluminum paste pigment (ASTM Specification D962–48T, Type II, Class A) per gallon of varnish meeting Federal Specification TTV81a, Type II, or the equivalent. Sufficient Prussian blue shall be added to permit detection of an incomplete application of the subsequent paint coat.

g.—After erection bare spots shall be touched up with zinc chromate primer followed by a touch-up coat of aluminum paint as specified in Specifications I–8e and I–8f.

h.—The completed structure shall be finished according to one of the following methods:

(1) One field coat of aluminum paint as specified in Specification I–8f, except that Prussian blue shall be omitted from the field coat.

(2) One or more field coats of alkyd base enamel pigmented to meet a desired color scheme.

PART III. EXPLANATION OF SPECIFICATIONS

Section A. Summary of Allowable Stresses

A–1. Basic Tensile Design Stress.—The basic tensile design stress of 22,000 lb per sq in. represents a factor of safety of 2.41 based on the specified tensile yield strength. This is a larger factor of safety with respect to yield strength than is ordinarily encountered in structural specifications. In selecting this rather large factor of safety on yield strength, the committee was influenced to a considerable extent by the fact that there is a smaller spread between yield strength and tensile strength in this aluminum alloy than is commonly encountered in structural steels.

A–8, A–9, and A–13. Allowable Stresses on Rivets.—The allowable shearing and bearing stresses on rivets were selected on the basis of the results of numerous shearing and bearing tests. The factors of safety used are greater than those used for most of the other allowable stresses.[2]

A–6, A–11, and A–12. Allowable Stresses in Pins.—The allowable bending, shearing, and bearing stresses on pins were selected to bear about the same relation to the corresponding properties of the material as is the case in standard steel specifications. It is not anticipated that any wide use of pins will be made in aluminum alloy structures but it is assumed that where they are used they will be of the same material as the structural members themselves, and that they would probably be obtained in the form of rolled rod, ASTM Specification B211–48T (CS41, heat treated and aged).

Section B. Column Design

B–1. Curves for Allowable Compressive Stresses in Axially Loaded Columns.—The curves in Fig. 1 are the tangent-modulus column curves with a factor of safety of 2.5 and with a cutoff at the basic allowable design stress of 22,000 lb per sq in. The formulas for all three curves can be written[4,5]

$$\frac{P}{A} = \frac{\pi^2 E_t}{2.5\left(\frac{kL}{r}\right)^2} \qquad (8)$$

in which:

P is the allowable load on the column, in pounds;

A is the gross cross-sectional area of the column, in square inches;

E_t is the tangent modulus taken from the "minimum" compressive stress-strain curve for the material at stress corresponding to $2.5(P/A)$, in pounds per square inch;

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[4] "Column Strength of Various Aluminum Alloys," by R. L. Templin, R. G. Sturm, E. C. Hartmann, and M. Holt, *Aluminum Research Laboratories Technical Paper No. 1*, Aluminum Co. of America, Pittsburgh, Pa., 1938.

[5] "Inelastic Column Theory," by F. R. Shanley, *Journal of the Aeronautical Sciences*, Vol. 14, 1947, pp. 261–268.

L is the length of the column, in inches;
r is the least radius of gyration of the column, in inches; and
k is a factor describing the end conditions as defined in Fig. 1.

For values of slenderness ratio, L/r, greater than 72, the formula for the partial restraint curve in Fig. 1 reduces to

$$\frac{P}{A} = \frac{74{,}000{,}000}{\left(\frac{L}{r}\right)^2} \quad \text{.......................(9)}$$

B–7. Formulas for Combined Compression and Bending.—Eq. 1, which applies to bending in the direction of the applied bending moment, takes into account the additional bending moment due to the deflection of the column.[6]

Eq. 2, covering failure by buckling normal to the plane of the bending forces, is a simplified design formula which is conservative compared to test results and to the theoretical solution for this case. A theoretical solution was proposed by J. N. Goodier,[7] in 1942; and test results have been reported[8] by H. N. Hill, Assoc. M. ASCE, and J. W. Clark, Jun. ASCE.

B–8. Formula for Transverse Shear on Columns.—Eq. 3a is based on the transverse component of the column load at the point of maximum slope of the column in its deflected position. A derivation by Mr. Hartmann has been published elsewhere.[6]

Section C. Curve for Allowable Compressive Stress in Beam and Girder Flanges

C–1.—The curve in Fig. 2 is based on the theoretical solution for the critical bending moment in the flange of I-beams as given by S. Timoshenko.[9] It represents a factor of safety of 2.5 applied to the theoretical solution for beams subjected to a uniform bending moment. It is assumed that at the ends of the laterally unsupported length there is partial restraint against rotation about a vertical axis and complete restraint against lateral displacement or rotation about a horizontal axis parallel to the web. The part of the curve for values of $\frac{L}{\sqrt{B/S_c}}$ greater than 27.5 is based on elastic action, whereas the remainder is simply an extension of the same formula using tangent modulus rather than initial modulus. The curve has a cutoff at the basic allowable design stress of 22,000 lb per sq in. It is important to note that the term L is defined as "laterally unsupported length of compression flange," which is not necessarily the same as the span of the beam or the girder. The case of uniform bending moment has been used in setting up Fig. 2 because it is a good approxi-

[6] Discussion by E. C. Hartmann of "Rational Design of Steel Columns," by D. H. Young, *Transactions*, ASCE, Vol. 101, 1936, pp. 475–481.

[7] "Torsional and Flexural Buckling of Bars of Thin-Walled Open Section Under Compressive and Bending Loads," by J. N. Goodier, *Transactions*, A.S.M.E., Vol. 64, 1942, pp. A-103-A-107.

[8] "Lateral Buckling of Eccentrically Loaded I-Section Columns," by H. N. Hill and J. W. Clark, *Proceedings Separate*, ASCE (publication pending).

[9] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., 1936.

mation of conditions frequently encountered in actual design, and because it is somewhat more conservative than many of the other cases that might have been selected.

For values of $\frac{L}{\sqrt{B/S_c}}$ greater than 27.5 the curve in Fig. 2 may be represented by the formula:

$$f_B = \frac{10{,}900{,}000}{\left(\frac{L}{\sqrt{B/S_c}}\right)^2} \qquad (10)$$

The curve in Fig. 2 is based on a theoretical solution applicable only to I-beams having cross sections symmetrical about both axes. The modified interpretation of the term I_1 indicated in Fig. 2, however, permits the curve to be used without serious error for beams and girders having one flange differing in lateral stiffness from the other. It should be used with caution in cases of beams and girders which are unsymmetrical by a considerable margin. (In connection with this subject several supplementary references [7,10,11,12,13] will be of interest.)

Section D. Curves for Design of Flat Plates, Legs and Webs

The curves of Figs. 3, 4, and 5 are based on values of critical stress compiled by Mr. Hill in 1940.[14] Partial restraint along the supported edges and loaded edges was assumed in all cases except for the supported edge in Fig. 3 which was considered simply supported. Parts of the curves that represent critical buckling stresses above the elastic range are computed by using the tangent modulus instead of the modulus of elasticity, a procedure which is known to be conservative when applied to problems of plate buckling.[15] A factor of safety of 2.5 against critical buckling has been used in all three charts and in all cases the curves have a cutoff at the basic allowable design stress of 22,000 lb per sq in.

When a flat plate, leg, or web is built in along one or both edges to other parts of a compression member which offer partial edge restraint, the local buckling of the plate, leg, or web does not precipitate collapse of the member as a whole as it probably would in the case of a single-angle strut. For this reason it is proper to permit a decreased factor of safety against local buckling in such cases if suitable precautions are taken to avoid collapse. Step c

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[10] "The Lateral Instability of Unsymmetrical I-Beams," by H. N. Hill, *Journal of the Aeronautical Sciences*, Vol. 9, 1942, pp. 175–180.

[11] "The Lateral Stability of Equal-Flanged Aluminum-Alloy I-Beams Subjected to Pure Bending," by C. Dumont and H. N. Hill, *Technical Note No. 770*, National Advisory Committee for Aeronautics, Washington, D. C., 1940.

[12] "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," by George Winter, *Transactions*, ASCE, Vol. 108, 1943, pp. 247–268.

[13] "Strength of Beams as Determined by Lateral Buckling," by Karl de Vries, *ibid.*, Vol. 112, 1947, pp. 1245–1320.

[14] "Chart for Critical Compressive Stress of Flat Rectangular Plates," by H. N. Hill, *Technical Note No. 773*, National Advisory Committee for Aeronautics, Washington, D. C., 1940.

[15] "Determination of Plate Compressive Strengths," by George J. Heimerl, *Technical Note No. 1480*, National Advisory Committee for Aeronautics, Washington, D. C., 1947.

of Specification D-2 provides a simple method for accomplishing this result by introducing the well known "effective width" concept. After a plate, leg, or web buckles, a part of its area is considered to be ineffective in supporting load, whereas a strip along each supported edge is considered still fully effective in working with the supporting material to which it is attached. The formula (Eq. 4) for effective width used in step c of Specification D-2 is based on the simple assumption that the total load supported by the effective width of a plate, leg, or web is never greater than the load which the full unsupported width was carrying at the critical buckling stress. In terms of allowable stress this can be expressed as $b_e f_c = b f_1$, which is easily rewritten in the form shown in Eq. 4. This method of calculating effective width is generally more conservative than other accepted methods.[16,17,18,19]

Section E. Curves for Allowable Shear Stress in Webs

E-1.—The values of allowable stress in Fig. 6 are obtained by applying a factor of safety of 2 to the critical shear buckling stresses for flat plates with the edges about half way between the fixed and hinged conditions.[13,20,21]

Those parts of the curves of Fig. 6 which represent critical buckling stresses above the elastic stress range are computed from formulas for elastic buckling with the tangent modulus substituted for the modulus of elasticity. For a given value of critical shear stress, the tangent modulus is that corresponding to an axial stress equal to $\sqrt{3}$ times the shear stress.[22] As in the case of compressive buckling of flat plates, the tangent modulus is conservative.

For values of allowable stress below 10,400 lb per sq in., the curves of Fig. 6 may be represented by the formula:

$$f_a = \frac{35{,}000{,}000}{(b/t)^2}\left[1 + 0.75\left(\frac{b}{a}\right)^2\right] \dots\dots\dots\dots\dots (11)$$

Section F. Plate Girder Design

F-6. Curves for Allowable Longitudinal Compressive Stress in Webs of Girders.—The curve in Fig. 7 for girders with no horizontal stiffeners is based on the critical buckling stress for rectangular flat plates under pure bending in the plane of the plate. Partial restraint is assumed at the toes of the flanges (about half way between the solution given by Mr. Timoshenko for the case of a plate simply supported on all four edges[13] and the solution of K. Nolke

[16] "The Strength of Thin Plates in Compression," by Theodor von Kármán, Ernest E. Sechler, and L. H. Donnell, *Transactions*, A.S.M.E., Vol. 54, 1932, pp. 53–57.

[17] "The Apparent Width of the Plate in Compression," by Karl Marguerre, *Technical Memorandum No. 833*, National Advisory Committee for Aeronautics, Washington, D. C., 1937.

[18] "Strength of Thin Steel Compression Flanges," by George Winter, *Transactions*, ASCE, Vol. 112, 1947, pp. 527–576.

[19] "Performance of Thin Steel Compression Flanges," by George Winter, preliminary publication, 3d Cong. of the International Assn. for Bridge and Structural Engrs., Liege, Belgium, 1948.

[20] "Formulas for Stress and Strain," by Raymond J. Roark, McGraw-Hill Book Co., Inc., New York, N. Y., 1938.

[21] "Observations on the Behavior of Aluminum Alloy Test Girders," by R. L. Moore, *Transactions*, ASCE, Vol. 112, 1947, pp. 901–920.

[22] "Critical Shear Stress of an Infinitely Long Plate in the Plastic Region," by Elbridge Z. Stowell, *Technical Note No. 1681*, National Advisory Committee for Aeronautics, Washington, D. C., 1948.

for a plate with the loaded edges simply supported and the other two edges fixed.[23]

The curve in Fig. 7 for girders with a single horizontal stiffener is based on the critical buckling stress given by Mr. Timoshenko for plates simply supported on all four edges under combined bending and axial stress in the plane of the plate.[13] The simple support condition is used for this case because the horizontal stiffener would provide comparatively little restraint against rotation. The location of the horizontal stiffener shown in the sketch in Fig. 7 is chosen so that the parts of the plate above and below the stiffener will buckle at approximately the same load.

A factor of safety against buckling of 1.5 was used for the curves of Fig. 7. Although this factor of safety is not as large as some used elsewhere in these specifications, it is considered adequate in this instance since tests have shown that the critical bending stress for girder webs may be considerably exceeded without affecting the load carrying capacity of the girder.[21,23] Use of Fig. 7, however, will prevent buckling from occurring at design stresses.

The curves of Fig. 7 may be represented by the following formulas: No horizontal stiffener—

$$f_a = \frac{200{,}000{,}000}{\left(\frac{h}{t}\right)^2} \qquad \text{(12a)}$$

and single horizontal stiffener—

$$f_a = \frac{800{,}000{,}000}{\left(\frac{h}{t}\right)^2} \qquad \text{(12b)}$$

The curves are cut off at the basic allowable design stress of 22,000 lb per sq in.

F–8 and F–9. Curves for Spacing and Moment of Inertia of Vertical Stiffeners.—The curves for determining stiffener spacing, in Fig. 8, are merely replots of the data of Fig. 6. The curves of I_s/t^4 in Fig. 8 represent the following formulas:[1] When $\frac{s}{h} \leqq 0.4$—

$$\frac{I_s}{t^4} = 6.13\,\frac{h}{t} \qquad \text{(13a)}$$

and, when $\frac{s}{h} > 0.4$—

$$\frac{I_s}{t^4} = \frac{\left(\frac{s}{h}\right)^2 + 0.625}{5\left(\frac{s}{h}\right)^4}\,\frac{h}{t} \qquad \text{(13b)}$$

[23] "Buckling of Webs in Deep Steel I-Girders," by Georg Wastlund and Sten G. A. Bergman, rept. of investigation made at the Royal Inst. of Technology, Stockholm, Sweden, 1947.

F-10. Formula for Moment of Inertia of Stiffeners at Points of Bearings.—Eq. 5 simply states that the moment of inertia of a stiffener at a point of bearing should be equal to the sum of the moment of inertia required to resist the tendency of the web to buckle and the moment of inertia required for the stiffener to carry the bearing load as a column with length equal to the height of the web.

F-11. Formula for Radius of Gyration of Horizontal Stiffeners.—Eq. 6, for the radius of gyration of horizontal stiffeners, is the same as that given by Mr. Moisseiff in 1940.[1] It is based on the results of the theoretical solutions for the stability of rectangular plates with one horizontal stiffener, subjected to pure bending in the plane of the plate.

Section H. Miscellaneous Design Rules

H-2 Formula for Slenderness Ratio of Tension Members.—Eq. 7 is designed to yield slenderness ratios in agreement with values generally accepted for tension members, at the same time taking into account the fact that the higher the minimum tensile stress on the member the less tendency there will be for the member to bend or sway.

H-4. Curves of Allowable Tensile Stress on Net Section for Various Numbers of Repetitions of Load Application.—The curves in Fig. 9 are plotted from the results of fatigue tests conducted at the Aluminum Research Laboratories of the Aluminum Company of America at New Kensington, Pa., on 14S-T6 butt joints with double straps joined with eight cold-driven 5/8-in. A17S-T3 rivets. The type of testing equipment and specimen (Type M1) used are illustrated in a paper by R. L. Templin,[24] M. ASCE, in 1939, and a paper by Mr. Hartmann, J. O. Lyst, and H. J. Andrews,[25] Jun. ASCE, in 1944.

A factor of safety of 1.2 has been applied to the test data and all curves are cut off at the basic allowable design stress of 22,000 lb per sq in. The right-hand part of the diagram is largely based on extrapolation of the data, but this is not considered to be a serious matter since the design of most members in this range will be governed primarily by Specification H-1 rather than by fatigue considerations.

Respectfully submitted,

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Committee of the Structural Division on Design in Lightweight Structural Alloys

November 1, 1949

[24] "Fatigue Machines for Testing Structural Units," by R. L. Templin, *Proceedings*, A.S.T.M., Vol. 39, 1939, pp. 711-722.

[25] "Fatigue Tests of Riveted Joints," by E. C. Hartmann, J. O. Lyst, and H. J. Andrews, *Wartime Report W55*, National Advisory Committee for Aeronautics, Washington, D. C., 1944.

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